Contents

Chapter 4 Open Channel Flow

4-1 General

An open channel is a watercourse, which allows part of the flow to be exposed to the atmosphere. This type of channel includes rivers, culverts, stormwater systems that flow by gravity, roadside ditches, and roadway gutters. Open channel flow design criteria are used in several areas of transportation design including:

1. River channel changes.
2. Streambank protection.
3. Partially full-flow culverts.
4. Roadside ditches.
5. Bridge design.
6. Down Stream Analysis
7. Weirs for irrigation.

Proper design requires that open channels have sufficient hydraulic capacity to convey the flow of the design storm. In the case of earth lined channels or river channels, bank protection is also required if the velocities are high enough to cause erosion or scouring.

River stabilization maybe necessary for highly erosive, high-energy rivers, to help the river to dissipate some of its energy and stabilizes the river banks and channel bottom. There are several rock structures that can be used to dissipate energy, this chapter will focus on two types: bank barbs and drop structures. The success of the rock structures or rock bank protection is dependent on the ability of the rock to withstand the forces of the river and thus it is of great importance to properly size the rocks used. The methodology for sizing rocks used in river stabilization is described in section 4-6.

The flow capacity of a culvert is often dependent on the channel up and downstream from that culvert. For example, the tailwater level is often controlled by the hydraulic capacity of the channel downstream of the culvert. Knowing the flow capacity of the downstream channel, open channel flow equations can be applied to a typical channel cross section to adequately determine the depth of flow in the downstream channel. This depth can then be used in the analysis of the culvert hydraulic capacity and is further discussed in section 4-4.

Shallow grass lined open channels can contribute to the cleaning of stormwater runoff before it reaches a receiving body. When possible, the designer should route stormwater runoff through open, grass lined ditches, also known as biofiltration swales. When road silts are permitted to settle out, they usually take with them a significant portion of other pollutants. The difference between a ditch and a bioswale is defined in section 4-3 along with the design criteria for ditches. The design criteria for biofiltration swales can be found in Chapter 5 of Washington State Department of Transportation (WSDOT) *Highway Runoff Manual*.

A downstream analysis identifies and evaluates the impacts, if any, a project will have on the hydraulic conveyance system downstream of the project site. The analysis should be broken into three sections: 1) Review of Resources; 2) Inspection of Drainage Conveyance Systems in the Site Area; and 3) Analysis of offsite effects. See section 4-7 of this chapter and the Hydraulic Report Outline in Chapter 1.

Measurement of flow in channels can be difficult because of the non-uniform channel dimensions and variations in velocities across the channel. Weirs allow water to be routed through the structure of known dimension, permitting flow rates to be measured as a function of depth of flow through the structure. Weirs for irrigation ditches are discussed in section 4-8.

4-2 Determining Channel Velocities

In open channel flow, the volume of flow and the rate at which flow travels are useful in designing the channel. For the purposes of this manual, the determination of the flow rate in the channel, also known as discharge, are based on the continuity of flow equation or equation 4-1 below. This equation states that the discharge (Q) is equivalent to the product of the channel velocity (V) and the area of flow (A).

$$Q = V A \tag{4-1}$$

Where: Q = discharge, cfs (m^3/s)

V = velocity, ft/s (m/s)

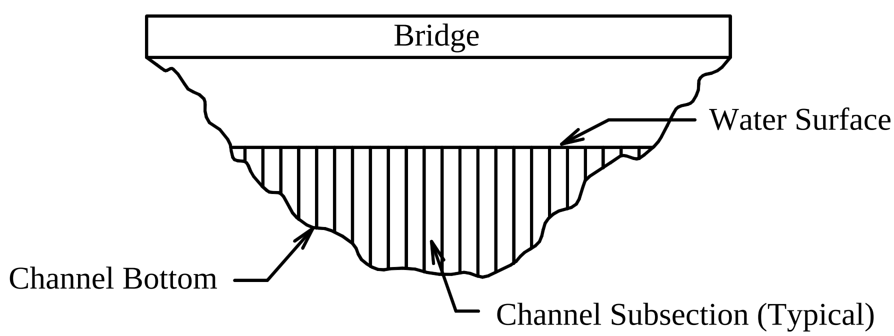
A = flow area, ft^2 (m^2)

In some situations, the flow area of a channel is known. If it is not, the flow area must be calculated using an iterative procedure described in Section 4-2.2. Computer programs and charts from FHWA Hydraulic Design Series No. 3 are also available for determining channel geometry or velocities. Channel velocities can either be measured or calculated using Manning's Equation as described below.

4-2.1 Field Measurement

Because channel velocities are used in determining flow rates, measurements of the channel velocity taken during periods of high flow are of most interest. The designer needs to consider the high flows and ensure that the channel design can provide the required capacity. The velocity can be estimated from field measurements by using one of the following three methods. The first two methods require the use of a current meter to measure velocities at any given depth in the channel.

Method 1 - The first method uses surveyed cross sections of the river. At a given cross section, the section is divided into subsections (up to 10 or 20 subsections for best accuracy) as shown in Figure 4-2.1. A change in depth or a change in ground cover is the best place to end a subsection. The current meter is used at each subsection to measure the velocities at 0.2 times the channel depth and at 0.8 times the channel depth. For example, if the channel was only one foot deep in the first subsection, the current meter should be lowered into the water to 0.2 ft from the channel bottom and used to read the velocity at this location. The designer would then raise the current meter to 0.8 ft from the channel bottom and read the velocity at that location. The velocity of that subsection of the river is the average of these two values. The process is repeated for each of the subsections.



Determining Velocities by Subsections

Figure 4-2.1

Method 2 – The second method requires, contour maps or surveyed cross sections of the river. Similar to the first method, the cross section of the river is divided into subsections. However, in the second method, the velocity is only measured at a distance from the channel bottom equivalent to 0.4 times the channel depth. This is considered to be the average velocity for that subsection of the river. A reading is taken at each subsection. This method is slightly less accurate than Method 1.

Method 3 - The third method is the least accurate of the three procedures. At the point of interest, the designer should measure the velocity at the surface of the stream. If no current meter is available, throwing a float in the water can do this and observing the time it takes to travel a known distance. The surface velocity is the known distance divided by the time it took to travel that distance. The average velocity is generally taken to be 0.85 times this surface velocity.

Once the velocity of each subsection is measured, the flow rate for each of the subsections is calculated as the product of the area of the subsection and its measured velocity. Summing the flow rates for each subsection will determine the total flow rate, or hydraulic capacity at this cross section of the river.

4-2.2 Manning's Equation

When actual stream velocity measurements are not available, the velocity can be calculated using Manning's Equation. Manning's Equation is an open channel flow equation used to find either the depth of flow or the velocity in the channel where the channel roughness, slope, depth, and shape remain constant (Steady Uniform Flow). The depth of flow using Manning's Equation is referred to as the normal depth and the velocity is referred to as the normal velocity.

The geometry involved in solving Manning's Equation can be complex and consequently, a direct mathematical solution for some channel shapes is not possible. Instead, a trial and error approach may be necessary. Various design tables are available to assist in these solutions as well as several personal computer programs. Information regarding sample programs is available from the Head Quarters (HQ) Hydraulics Office.

4-2.2.1 Hand Calculations

The solution for velocity in an open channel must conform to the following formula:

$$V = \frac{1.486}{n} R^{2/3} \sqrt{S} \; (\text{English Units}) \qquad (4\text{-}2)$$

$$V = \frac{1}{n} R^{2/3} \sqrt{S} \; (\text{Metric Units})$$

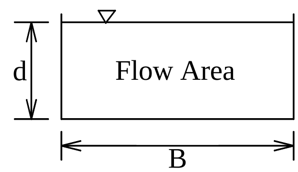
Where:

V = Mean velocity in channel, ft/s (m/s)

n = Manning's roughness coefficient (see Appendix 4-1)

S = Channel slope – steady and uniform flows occurs, ft/ft (m/m)

R = Hydraulic radius, ft (m)



R = A/WP (4-3)

A = Flow Area of the cross section of water, ft^2 (m^2)

See Figure 4-2.2.1 for additional area equations

WP = Wetted perimeter, ft (m)

WP = d + B + d (4-4)

See Figure 4-2.2.1 for additional WP equations

The hydraulic radius is the ratio of flow area to the wetted perimeter where the wetted perimeter is the length of channel cross section that is in contact with the water. For full flow circular pipes, the hydraulic radius is one-fourth the diameter of the pipe. In relatively flat, shallow channels, where B > 10d, the wetted perimeter can be approximated by the width of the channel. As a result, the hydraulic radius can be approximated as the depth of water, R $\cong$ d.

$$R = \frac{A}{WP} = \frac{Bd}{B + 2d} = \frac{Bd}{B} = d$$

When the depth of flow is known, the mathematical solution is simple. The section properties area (A) and wetted perimeter (WP) can be determined and put into the equation to find velocity (V).

The flow rate, or discharge can then be found by equation 4-1:

$$Q = VA$$

More frequently, the designer knows the discharge but the depth of flow in the channel must be determined. Since Manning's Equation cannot solve for the depth of a trapezoidal channel directly, a method of successive approximations must be used. The designer must estimate the depth, determine the section properties, and finally solve for the discharge. If the discharge so derived is too high, the designer must then revise the estimated depth downward and recalculate the discharge. This process is repeated until the correct discharge is found within sufficient accuracy (3 to 5 percent). This method can be time consuming. It is recommended that a programmable calculator or computer be used to aid in the computations.

Regardless of whether the depth is known or needs to be calculated, the designer must verify that the normal depth of the channel is either greater than or less than the critical depth of the channel as described in section 4-4 of this Chapter.

4-2.2.1.1 Examples - Manning's Equation using Hand Calculations

For the following hand calculation examples using Manning equations, designers should use Figure 4-2.2.1, Geometric Elements of Channel Sections.

Example 1

A trapezoidal channel with 1.75:1 side slopes and a 6.5 ft bottom width is flowing 4 ft deep. The channel has a bottom slope of 0.004 ft/ft for a distance of several hundred feet. What is the discharge of the riprap lined channel?

Since this is a small channel with riprap, the roughness coefficient of 0.040 is chosen from Appendix 4-1.

$$A = (b + ZD)D = [6.5\text{ft} + 1.75(4\text{ft})]4\text{ft} = 54\text{ft}^2 \tag{4-5}$$

$$WP = b + 2D\sqrt{1 + Z^2} = 6.5\text{ft} + 2(4)\sqrt{1 + 1.75^2} = 22.6\text{ft} \tag{4-4}$$

$$R = A/WP = 54\text{ft}^2/22.6\text{ft} = 2.4\,\text{ft} \tag{4-3}$$

$$V = \frac{1.486}{n}R^{2/3}\sqrt{S} = \frac{1.486}{0.04}(2.4)^{2/3}\sqrt{0.004} = 4.2\text{ft/s} \tag{4-2}$$

$$Q = VA = 4.2\text{ft/s}(54\text{ft}^2) = 226.8\text{cfs} \tag{4-1}$$

Example 2

How deep would the channel described above flow if the discharge is 600 cfs?

The designer needs to assume various depths and solve for Q using equation 4-1. It may be helpful to draw a graph to aid in choosing the next depth. Once a Q both below and above the given discharge, in this case 600 cfs, is determined the depth can be found using interpolation as shown below.

$$Q = VA \qquad (4\text{-}1)$$

Next substitute equation 4-2 in for the velocity and the appropriate area from Figure 4-2.2.1.

$$Q = ((b + ZD)D) \times (\frac{1}{n} R^{2/3} \sqrt{S})$$

Assumed D	**Calculated Q**
4 ft	226.8 cfs
6.6 ft	655.1 cfs
6.2 ft	581.0 cfs
6.4 ft	611.8 cfs

Interpolate for depth (d) at discharge 600 cfs:

1. Locate two discharge points, one above and one below 600 cfs, and note the depth.

   Q = 581.0cfs   d = 6.2ft

   Q = 611.8cfs   d = 6.4ft

2. When interpolation is used, it is assumed that there is a linear relationship between the points. In other words if a straight line was drawn, all 3 points (or discharge values Q) could be located on that line. If there is an unknown coordinate for one of the points, it can be found by finding the slope of the line, as shown below:

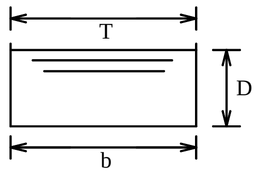
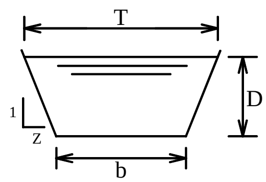
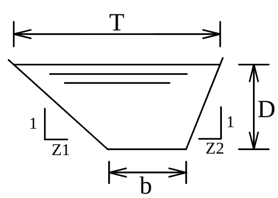
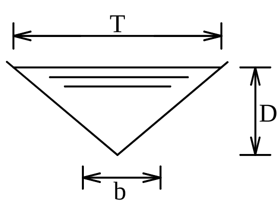
$$\frac{(6.4\text{ft} - 6.2\text{ft})}{\left(611.8\frac{\text{ft}^3}{\text{s}} - 581.0\frac{\text{ft}^3}{\text{s}}\right)} = 0.00649\frac{\text{ft}}{\frac{\text{ft}^3}{\text{s}}}$$

3. Once the slope is known, the depth can be determined at 600 cfs:

$$\left(600\frac{\text{ft}^3}{\text{s}} - 581.0\frac{\text{ft}^3}{\text{s}}\right) \times 0.00649\frac{\text{ft}}{\frac{\text{ft}^3}{\text{s}}} = 0.12$$

$$d = 6.2\text{ft} + 0.12\text{ft} = 6.32\text{ft}$$

4. Finally, the depth should be verified by rerunning the analysis at d=6.32ft to verify Q is 600cfs. Calculations accurate to ± 3 percent are sufficient.

Cross	Area, A (Equation 4-5)	Wetted Perimeter, WP (Equation 4-4)
 Rectangle	BD	B+2D
 Trapezoid (Equal side slopes)	(b+ZD)D	$b + 2D\sqrt{1+Z^2}$
 Trapezoid (unequal side slopes)	$\frac{D^2}{2}(Z_1 + Z_2) + Db$	$b + D(\sqrt{1+Z_1^2} + \sqrt{1+Z_2^2})$
 Triangle	ZD^2	$2D\sqrt{1+Z^2}$

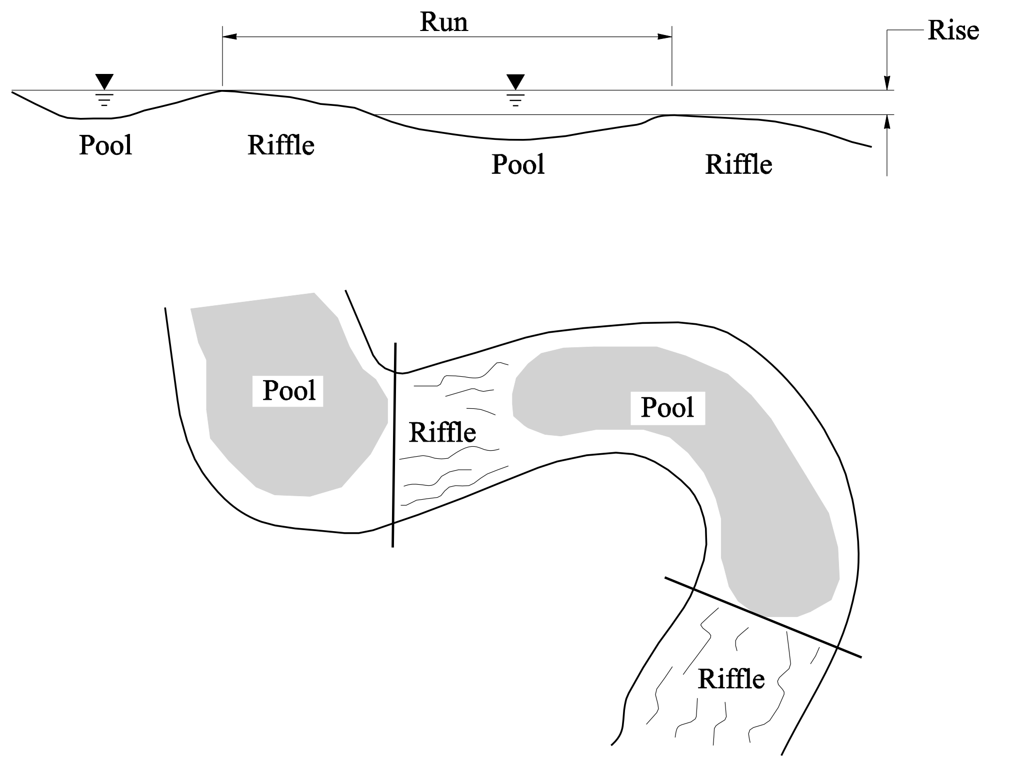
Reference: VT Chow “Open Channel Hydraulics” for a more complete table of geometric elements.

Geometric Elements of Channel Sections

Figure 4-2.2.1

4-2.2.2 Field Slope Measurements

By definition, slope is rise over run (or fall) per unit length along the channel centerline or thalweg. Slope is the vertical drop in the river channel divided by the horizontal distance measured along the thalweg of a specific reach. The vertical drop should be measured from the water surface at the top-of-riffle (end of pool) to the next top-of-riffle to get an accurate representation of the slope in that reach.



Field Slope Measurement

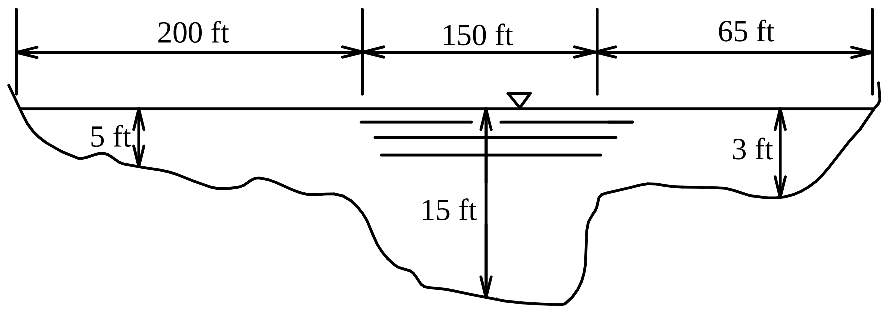
Figure 4-2.2.2

4-2.2.3 Manning's Equation in Sections

Manning's method by sections should be used when the channel is distinctly different from the overbank; varying depths and roughness values. Channels and flood plains have a common occurrence of this type. If an average depth or Manning's value were used for this situation instead the results would be less accurate. The following example illustrates this situation.

4-2.2.3.1 Example Manning's Equation in Sections

Determine the velocity and discharge in each of the three subsections shown in Figure 4-2.2.3.1 The river slope is 0.003 ft/ft. The ground cover was observed during a field visit and the corresponding Manning's Roughness values were found in Appendix 4-1. Both the ground cover and Manning's values are noted below.



Manning's Equation in Sections

Figure 4-2.2.3.1

Subsections Method:	**Section 1**	**Section 2**	**Section 3**
Top Width, T	200 ft	150 ft	65 ft
Ground Cover	Trees	channel	Rock
Manning's Roughness	0.090	0.035	0.060
Flow Depth, D	5 ft	15 ft	3 ft
Area, A	1000 ft^2	2250 ft^2	195 ft^2
Hydraulic Radius, R	5 ft	15 ft	3 ft
Velocity, V	2.64 ft/s	14.3 ft/s	2.82 ft/s
Discharge, Q	2640 cfs	32175 cfs	550 cfs

The area for each section was found using the equation for a rectangle from Figure 4-2.2.1. The Hydraulic Radius was set equal to the depth, as noted in section 4-2.2.1 this can be done when the width of the channel is more than 10 times the depth. Using equation 4-2 the velocity was determined and finally the discharge was found with equation 4-1.The total flow rate is equal to the sum of the discharges from each subsection or 35,363 cfs (912 m^3/s), which would be the correct value for the given information.

To attempt this same calculation using a constant roughness coefficient, the designer would have to choose between several methods, which take a weighted average of the n-values. Taking a weighted average with respect to the subsection widths or subsection area may appear to be reasonable, but it will not yield a correct answer. The subsection method shown above is the only technically correct way to analyze this type of channel flow. However, this application of Manning's Equation will not yield the most accurate answer. In this situation, a backwater analysis, described in

Section 4-4, should be performed. Notice that the weighted average n-value is difficult to choose and that the average velocity does not give an accurate picture as the first method described in Section 4-2.1 Field Measurement.

4-3 Roadside Ditch Design Criteria

Roadside ditches are generally located alongside uncurbed roadways with the primary purpose of conveying runoff away from the roadway. Ditches should be designed to convey the 10-year recurrence interval with a 0.5-foot freeboard and a maximum side slope of 2:1. The preferred cross section of a ditch is trapezoidal however a ‘V’ ditch can also be used where right of way is limited and or the design requirements can still be met. In those cases where the grade is flat, preventing adequate freeboard, the depth of channel should still be sufficient to remove the water without saturating the pavement subgrade. To maintain the integrity of the channel, ditches are usually lined with grass, however this type of lining is only acceptable for grades up to 6% and with a maximum velocity of 5 feet per second. For higher velocities and channel slopes, more protective channel linings are required; see *HDS #4 Introduction to Highway Drainage* or section 9-33 of the *Standard Specifications* for more information.

Ditches should not be confused with Biofiltration Swales. In addition to collecting and conveying drainage, swales also provide runoff treatment by filtering out sediment. See Chapter 5 of the WSDOT *Highway Runoff Manual* for design guidance for Biofiltration Swales.

4-4 Critical Depth

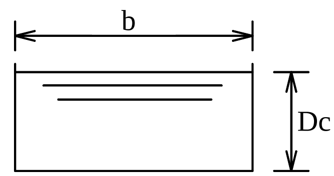
Before finalizing a channel design, the designer must verify that the normal depth of a channel (see section 4-2.2) is either greater than or less than the critical depth. Critical depth is the depth of water at critical flow, a very unstable condition where the flow is turbulent and a slight change in the specific energy, the sum of the flow depth and velocity head, could cause a significant rise or fall in the depth of flow. Critical flow is also the dividing point between the subcritical flow regime (tranquil flow), where normal depth is greater than critical depth, and the supercritical flow regime (rapid flow), where normal depth is less than critical depth.

Critical flow tends to occur when passing through an excessive contraction, either vertical or horizontal, before the water is discharged into an area where the flow is not restricted. A characteristic of critical depth flow is often a series of surface undulations over a very short stretch of channel. The designer should be aware of the following areas where critical flow could occur: culverts, bridges, and near the brink of an overfall.

A discussion of specific energy is beyond the scope of this manual. The designer should refer to any open channel reference text for further information. Critical depth can be found by the following formulas and demonstrated in the examples that follow:

1. Rectangular Channel

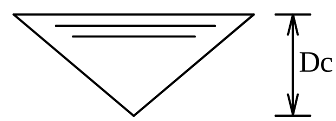
$$D_C = \left[\frac{C_1 Q}{b}\right]^{2/3} \qquad (4\text{-}6a)$$



Where C_1 = is 0.176 (English units) or 0.319 (metric units)

2. Triangular Channel

$$D_C = C_2\left[\frac{Q}{Z_1 + Z_2}\right]^{2/5} \qquad (4\text{-}6b)$$

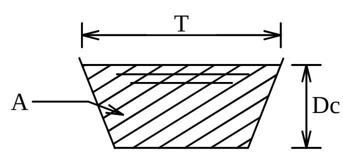


Where C_2 = is 0.757 (English units) or 0.96 (metric units)

3. Trapezoidal Channel

A trial and error or successive approximations approach is required with equation 4-7a when Dc is unknown:

$$Q = \left[\frac{gA^3}{T}\right]^{1/2} \qquad (4\text{-}7a)$$



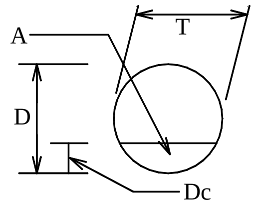
Where g = is the gravitational constant, 32.2 ft/s^2 (English units) or 9.81 m/s^2 (metric units)

A = can be found using equation 4-5 in Figure 4-2.2.1

4. Circular Shaped Channel

As with equation 4-7a, a successive approximation approach is required for equation 4-7b, when solving for Dc.

$$Q = \left[\frac{gA^3}{T}\right]^{1/2} \qquad (4\text{-}7b)$$



Where g = is the gravitational constant, 32.2 ft/s2 (English units) or 9.81 m/s2 (metric units)

In lieu of the trial and error approach with equation 4-7b, designers can instead use equation 4-6c for an approximate solution:

$$D_c = C_3 \frac{Q^{0.5}}{D^{0.25}} \qquad (4\text{-}6c)$$

Where C3 = 0.42 (English units) or 0.562 (metric units)

4-4.1 Example Critical Depth in a Rectangular Channel

Find the critical depth in a rectangular channel 15ft bottom width and vertical sidewalls using equation 4-6a. The discharge is 600 ft ^{3}/s.

$$D_c = \left[\frac{C_1 Q}{b}\right]^{2/3} = \left[\frac{0.176\left(600\text{ft}^3/\text{s}\right)}{15}\right]^{2/3} = 3.67\text{ft}$$

4-4.2 Example Critical Depth in a Triangular Channel

Find the critical depth in a triangular shaped channel with 1.75:1 sideslopes using equation 4-6b. The discharge is 890ft ^{3}/s

$$D_c = C_2\left[\frac{Q}{Z_1 + Z_2}\right]^{2/5} = 0.757\left[\frac{890\text{ft}^3/\text{s}}{1.75 + 1.75}\right]^{2/5}\ 6.94\,\text{ft}$$

4-4.3 Example Critical Depth in a Trapezoidal Channel

Find the critical depth in a trapezoidal channel that has a 10ft bottom width and 2:1 side slopes for a discharge of 1200cfs. Use equation 4-7b to solve for Q using a trial and error approach with different depths. Repeat the process until Q is close to 1200 cfs. A programmable calculator is strongly recommended.

$$Q = \left[\frac{gA^3}{T}\right]^{1/2}$$

Assumed D (ft)	A (ft^2)	T (ft)	$Q = \left[\frac{gA^3}{T}\right]^{1/2}$
4	72	26	680
6	132	34	1476
5.2	106	30.8	1116
5.4	112.3	31.60	1201

The critical depth for the given channel and discharge is approximately 5.4 ft (1.65m).

4-4.3 Example Critical Depth in a Circular Shaped Channel

Find the critical depth for a 3.5ft diameter pipe flowing with 18cfs and then for 180cfs using equation 4-6c.

For 18 cfs:

$$D_C = C_3 \frac{Q^{0.5}}{D^{0.25}} = 0.42 \frac{(18cfs)^{0.5}}{(3.5ft)^{0.25}} = 1.3ft$$

For 180 cfs:

$$D_C = C_3 \frac{Q^{0.5}}{D^{0.25}} = 0.42 \frac{(180cfs)^{0.5}}{(3.5ft)^{0.25}} = 4.1ft$$

Note that 4.1ft is greater than the diameter and therefore has no significance for open channel. The pipe would be submerged and would act as an orifice instead of an open channel.

4-5 River Backwater Analysis

Natural river channels tend to be highly irregular in shape so a simple analysis using Manning's Equation, while helpful for making an approximation, is not sufficiently accurate to determine a river water surface profile. Per Chapter 1, Section 1-2 of this manual, the HQ Hydraulics Office is responsible for computing water surface profiles and has several computer programs to calculate the water surface profile of natural river channels. The computation of the water surface profile is called a backwater analysis. The purpose of this section is to state when a backwater analysis is necessary as well as to summarize the minimum design requirements for the analysis and provide the project office with a list of field information required for HQ Hydraulics to perform an analysis.

A backwater analysis is performed when designing a bridge that crosses a river designated as a FEMA regulatory floodway. WSDOT is required by federal mandate to design these bridges to accommodate the 100-year storm event. And it is desirable to maintain a 3'foot vertical clearance between the bottom of the bridge and the 100-year water surface elevation. The water surface elevations for the 100-year and 500-year water surface profiles should be shown on the plans.

A backwater analysis can also be useful in the design of culverts. Computing the water surface profile can help the designer determine if the culvert is flowing under inlet, or outlet control. For additional information about backwater analyses, see FHWA's Hydraulic Design Series No. 1, Hydraulics of Bridge Waterways. The region must provide the following information to the HQ Hydraulics Office to complete a river backwater analysis.

1. A contour map of the project site with 1 ft (0.25 m) or 2 ft (0.50 m) intervals is required. The map should extend from at least one bridge length downstream of the bridge to any point of concern upstream with a minimum distance upstream of two bridge lengths and two meander loops. The map should include all of the area within the 100-year flood plain. All bridge and unique attributes of the project area should be identified.

2. The Manning's roughness coefficients must be established for all parts of the river within the project area. HQ Hydraulics Office will need photographs of the channel bed and stream bank along the reach of interest to determine the appropriate channel roughness. Photos are especially important in areas where ground cover changes.

To prevent subsequent difficulties in the backwater analysis, the HQ Hydraulics Office should be contacted to determine the necessary parameters.

4-6 River Stabilization

The rivers found in Washington are still very young in a geological sense and will tend to move laterally across the flood plain from time to time until equilibrium is reached. Whenever a river is adjacent to a highway, the designer should consider the possible impacts of the river on the highway or bridge.

In a natural setting, a river is exposed to several channel characteristics, which help to dissipate some of its energy. Such characteristics include channel roughness, meanders, vegetation, obstructions like rocks or fallen trees, drops in the channel bottom, and changes in the channel cross section. The meander provides an additional length of channel, which allows the river to expend more energy for a given drop in elevation. Vegetation increases the roughness of the channel causing the flow to dissipate more of its energy in order to flow through it. The river utilizes both increased channel length from meanders and increased channel roughness from vegetation to dissipate some of its energy during high runoff periods. When a river overtops its banks, it begins to utilize its flood plains. The flow is either stored in the overbank storage provided by the flood plain or returns to the river downstream. Compared to the flow in the river, the flow returning to the river has been slowed significantly due to the increased roughness and travel length.

Inevitably, roadways are found adjacent to rivers because roadway construction costs are minimized when roadways are constructed through level terrain. At times, roadways built in the flood plain confine the river to one side of the roadway, reducing its channel length. At other times, rivers are confined to their channel to minimize flooding of adjacent properties. As a result, rivers are unable to utilize overbank storage areas. These two situations produce rivers that are highly erosive because the river can no longer dissipate the same amount of energy that was dissipated when the river was not confined to a certain area.

These highly erosive rivers have caused significant damages to the state’s highways and bridges. Many roadway embankments have been damaged and bridge piers have been undermined, leading to numerous road closures and high replacement costs. Due to the extensive flooding experienced in the 1990s, more attention has been given to stabilizing Washington Rivers and minimizing damages.

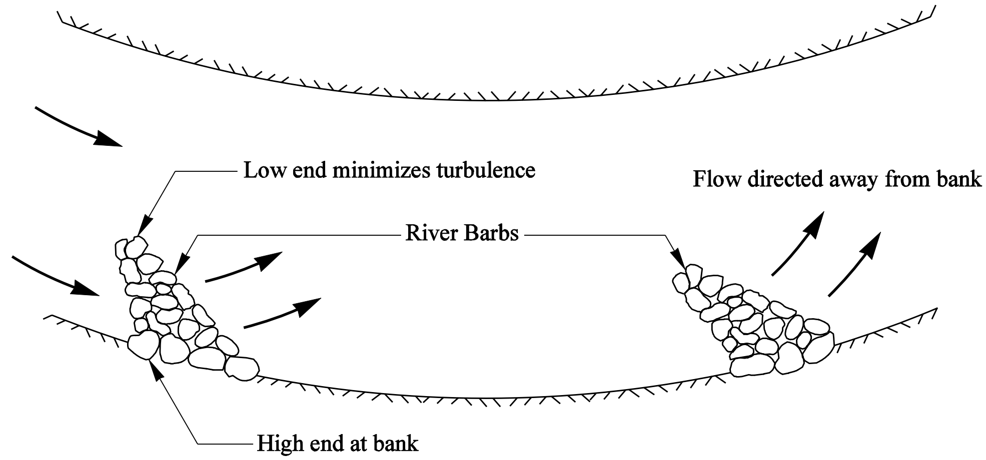
For highly erosive, high-energy rivers, structures constructed in the river’s channel are beneficial because they help the river to dissipate some of its energy and stabilize its banks and channel bottom. There are several rock structures that can be used to dissipate energy. Two structures described in the following sections include bank barbs and drop structures. Guide banks and spurs are other examples of in-channel rock structures. Detailed descriptions of guide banks and spurs are provided in the *Hydraulic Engineering Circular No. 20 — Stream Stability at Highway Structures* (http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm). When the use of these rock structures is not feasible, riprap bank protection can be used and is described further in Section 4-6.3. See Section 4-6.1 and Section 4-6.2 for feasible applications for bank barbs and rock drop structures. For further guidance on Barbs, designers can consult the following WSDOT research document: *Investigation of Flow and Local Scour Characteristics around a partially submerged permeable WSDOT Barb*, WA-RD581.1 Feb 2004

The success of the rock structures or rock bank protection is dependent on the ability of the rock to withstand the forces of the river. As a result, it is of great importance to properly size the rocks used for barbs, drop structures, and bank protection. Although the procedure for sizing the rocks used for barbs and drop structures are similar, riprap sizing for bank protection is not. The methodology for sizing rocks used in each of these structures is described in the individual sections.

For the purposes of this manual, river stabilization techniques include in-channel hydraulic structures only. Bioengineering is the combination of these structures with vegetation, or only densely vegetated streambank projects, which provide erosion control, fish habitat, and other benefits. The designer should consult WSDOT's *Design Manual* Soil Bioengineering Chapter for detailed information about bioengineering. Additionally, the Stream Habitat Restoration Guidelines (SHRG) provides guidance not just for stabilizing rivers, but also considering techniques that provide a natural stream restoration, rehabilitating aquatic and riparian ecosystems. (http://wdfw.wa.gov/hab/ahg/shrg/index.htm)

4-6.1 Bank Barbs

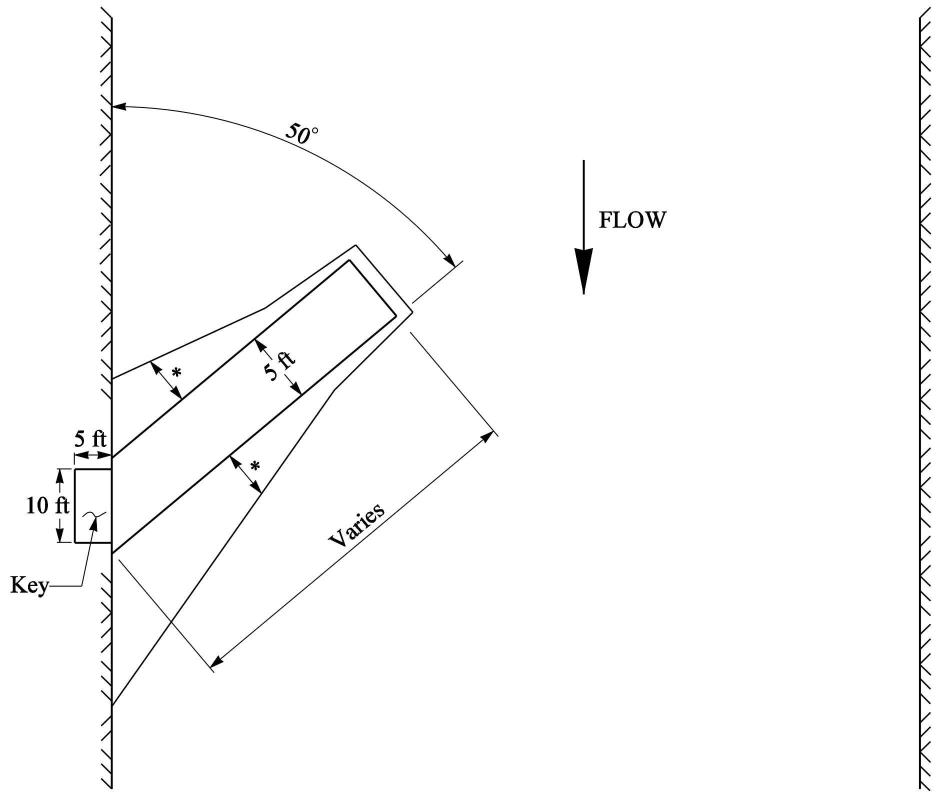
Riprap lined channels are very smooth hydraulically. As a result, the river takes the path of least resistance and the deepest part of the channel, or thalweg, is found adjacent to the riprap bank protection. With the thalweg immediately adjacent to the bank protection, scour occurs and the bank protection can be undermined if a toe is not sufficiently keyed into the channel bottom. In this case, it is necessary to shift the thalweg away from the bank and dissipate some of the river's energy to minimize the river's erosive capacity. This can be accomplished by using a bank barb: a trapezoidal shaped rock structure, which extends into the main flow of the river as shown in Figure 4-6.1.1. Since barbs tend to redirect water to the center of the stream, they encourage deposition between the barbs along the bank.



River Barb Typical Plan View

Figure 4-6.1.1

The bank barb should extend upstream one-third of the way into the bank full channel width or the mean channel width, at a 50-degree angle, as shown in Figure 4-6.1.2. This orientation will capture part of the flow and redirect it perpendicular to the downstream face of the barb. Generally, one barb can protect the length of bank equivalent to about four times the length of the barb perpendicular to the bank. This length of protection is centered about the barb such that two perpendicular barb lengths of bank upstream of the barb and two perpendicular barb lengths of bank downstream of the barb are protected.



River Barb Schematic

Figure 4-6.1.2

The benefits of constructing bank barbs are numerous. The rock structure provides additional roughness to the channel, which slows the flow and helps to decrease its energy. This in turn will reduce the erosive capacity of the river and minimize impacts to roadway embankments and streambanks. They are cost effective since they are less expensive than the alternatives of constructing a wall or placing riprap along a long section of bank. Barbs also provide fish habitat, if habitat features such as logs and root wads are incorporated into the barbs. For more information regarding fish habitat, refer to Chapter 7.

The barbs redirect flow away from the bank minimizing the potential of slope failure. Their ability to redirect the flow can also be useful in training the river to stay within its channel instead of migrating laterally. The designer should consider minimizing river migration when a bridge spans the river. When a bridge is originally constructed, it is designed in such a way that the river flows through the center of the bridge opening. However, after several years, the river will more than likely migrate laterally, possibly endangering bridge piers or abutments because it now flows only along the left side or right side of the opening or it flows at an angle to the bridge.

Barbs are an effective tool both training the river to flow through the bridge opening while protecting the bridge abutments.

As effective as barbs are at redirecting flow, there are a few situations where barbs should not be used. For rivers with large bed load (i.e., large quantities of sediments, or large size rocks), barbs may not be as effective at stabilizing the river. Barbs encourage sediments to settle out of the water because they intercept flow and slow it down. If a river has large quantities of sediments, a lot of sediment will tend to settle out upstream and downstream of the barb. The barb will lose its geometric structure and go unnoticed by the river. If the sediments carried downstream by the river are large in size, the barbs could be destroyed from the impact of large rocks or debris.

Barbs may also be ineffective in rivers that flow in a direction other than parallel to the streambank. A barb would not be as effective in this situation because if the flow was at an angle to the streambank, the barb would intercept very little of the flow and thus provide very little redirection.

Three considerations should be taken into account when designing a barb: the size of rock to be used, its placement, and vegetation. For further design guidance, designers can consult *HEC 23 Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance* (http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm), or *Integrated Streambank Protection Guidelines (ISPG)* (http://wdfw.wa.gov/hab/ahg/ispgdoc.htm) or the Region Hydraulics Engineer.

4-6.1.1 Riprap Sizing for Bank Barbs

The procedure for determining the size of rock needed for a barb can be based on tractive force theory, channel slope, and maximum permissible depth of flow. Tractive force theory is the shear stress exerted by the flow on the channel perimeter, where shear stress is equivalent to the product of channel slope, depth of flow, and the density of water. As any of these factors increase, shear stress increases, and the size of rock necessary to withstand the force of the water will increase. The rock used in the barb must be large enough in both size and weight to resist the force of the water. If the rock is not large enough to withstand the shear stress exerted by the flow, it will be washed downstream.

Assuming that the normal density of water is 62.4 lbf/ft^3 (9810 N/m^3) and the specific gravity of rock riprap is 2.65, a relation between rock size and shear stress as related to the product of depth times slope is provided below. Once the average channel slope and depth of flow for the 100-year event is known, the designer can determine the riprap gradation to be used. If the product of slope times flow depth falls between riprap gradations, the larger gradation should be used.

The riprap sizing procedure for bank barbs is not the same procedure used for riprap bank protection. In the case of a barb, the rock is located within the river channel and fully exposed to the flow of the river. The riprap sizing is based on charts relating shear stress to sediment size from *Hydraulic Engineering Circular No. 15 - Design of Roadside Channels with Flexible Linings* and *Hydraulic Engineering Circular No. 11- Use of Riprap for Bank Protection* (http://www.fhwa.dot.gov/engineering/hydraulics/software.cfm). For riprap bank protection, the rock is located along the streambanks with the flow being parallel to the bank. The size of rock required for bank protection will be smaller since its entire surface is not exposed to the flow. Riprap sizing for bank protection is described in Section 4-6.3.

	D_{50}		Slope Times Flow Depth	
Riprap Gradation	**English (ft)**	**Metric (m)**	**English (ft)**	**Metric (m)**
Spalls	0.5	0.15	0.0361	0.011
Light Loose Riprap	1.1	0.32	0.0764	0.0233
Heavy Loose Riprap	2.2	0.67	0.1587	0.0484
1 Meter D50 (Three Man)[1]	3.3	1	0.2365	0.0721
2 Meter D50 (Six Man)[1]	6.6	2	0.5256	0.1602

1. See Standard Specification Section 9-13.7(1).

Riprap Sizing for In-Channel Structures

Figure 4-6.1.3

4-6.1.1.1 Example Riprap Sizing for River Barb

Determine the riprap gradation required for a river barb in a reach of river with a channel slope of 0.0055ft/ft and flow depth of 16.4ft.

Slope Times Flow Depth = S x d (4-8)

Where: S = slope of the channel

d = flow depth

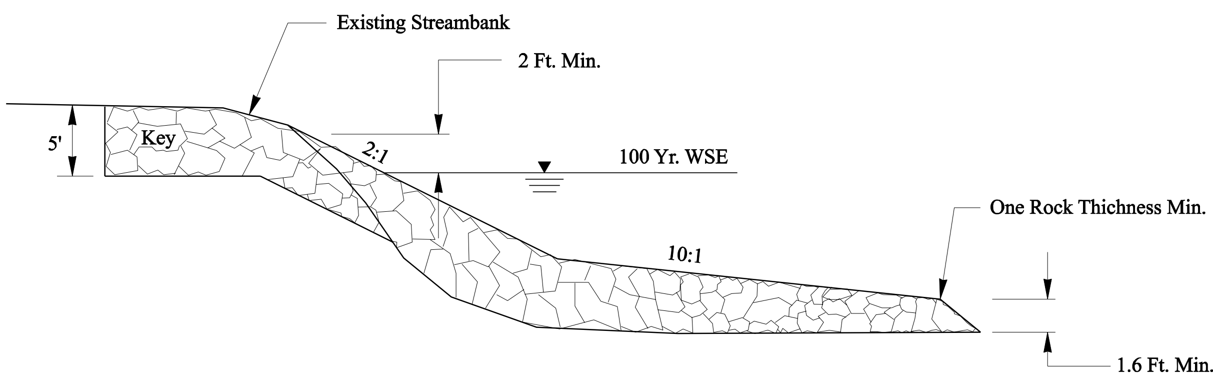
Slope Times Flow Depth = 0.0055 x 16.4= 0.0902

Next, use Figure 4-6.1.3 to determine the Riprap Size. Since the Slope Times the Flow Depth falls between light loose and heavy loose riprap gradations, the larger gradation or heavy loose riprap should be used.

4-6.1.2 Riprap Placement for Bank Barbs

When placing the rocks, the larger rocks should be used to construct the base with the rock's longest axis pointed upstream. Smaller rocks can then be used to fill in the voids. The rocks used in the barb must be well graded to ensure interlocking between rocks. The interlocking mechanism is as important as the sizing of the rock. As long as the rocks used in the barb interlock, the barb acts as one entire unit and is better at resisting the shear stress exerted by the flow.

It is essential that the rocks used to form the downstream face are the larger rocks in the riprap gradation and securely set on the channel bottom. The larger rocks along the downstream face provide a base or foundation for the barb as these rocks are subjected to both the forces of the flow and the rocks along the upstream face of the barb. It is also very important to extend a key to the top of the bank or at least two foot above the 100-year flood elevation, see Figure 4-6.1.4. If the flow of water is allowed to get behind the key, the river will take the path of least resistance and the existing stream bank that the barb was tied into will erode. The barb will become an ineffective riprap island if not washed downstream.



River Barb Typical Cross Section

Figure 4-6.1.4

4-6.1.3 Vegetation

Vegetation is also a key factor for bank protection. Any land that has been cleared and is adjacent to a river is very susceptible to erosion. Establishing vegetation provides a root system, which can add to the stability of the bank. Plantings also add roughness to the channel slowing the flow. The erosive capacity of the river is reduced for a minimal cost as the energy is dissipated.

The designer should be aware that although vegetation provides some benefits as mentioned above, these benefits are not immediate. There is some risk involved in losing the plantings to a flood before it has time to establish itself and take root. Under favorable conditions, plantings such as willow cuttings and cottonwoods can establish their root systems within a year. Willow cuttings are recommended because of their high survival rate and adaptability to the many conditions specific to typical highway project sites. Cottonwoods are recommended for their extensive root system, which can provide some streambank stability. For detailed information regarding planting type and spacing, the designer should contact the regional landscape architecture office or HQ Roadside and Site Development Services Unit.

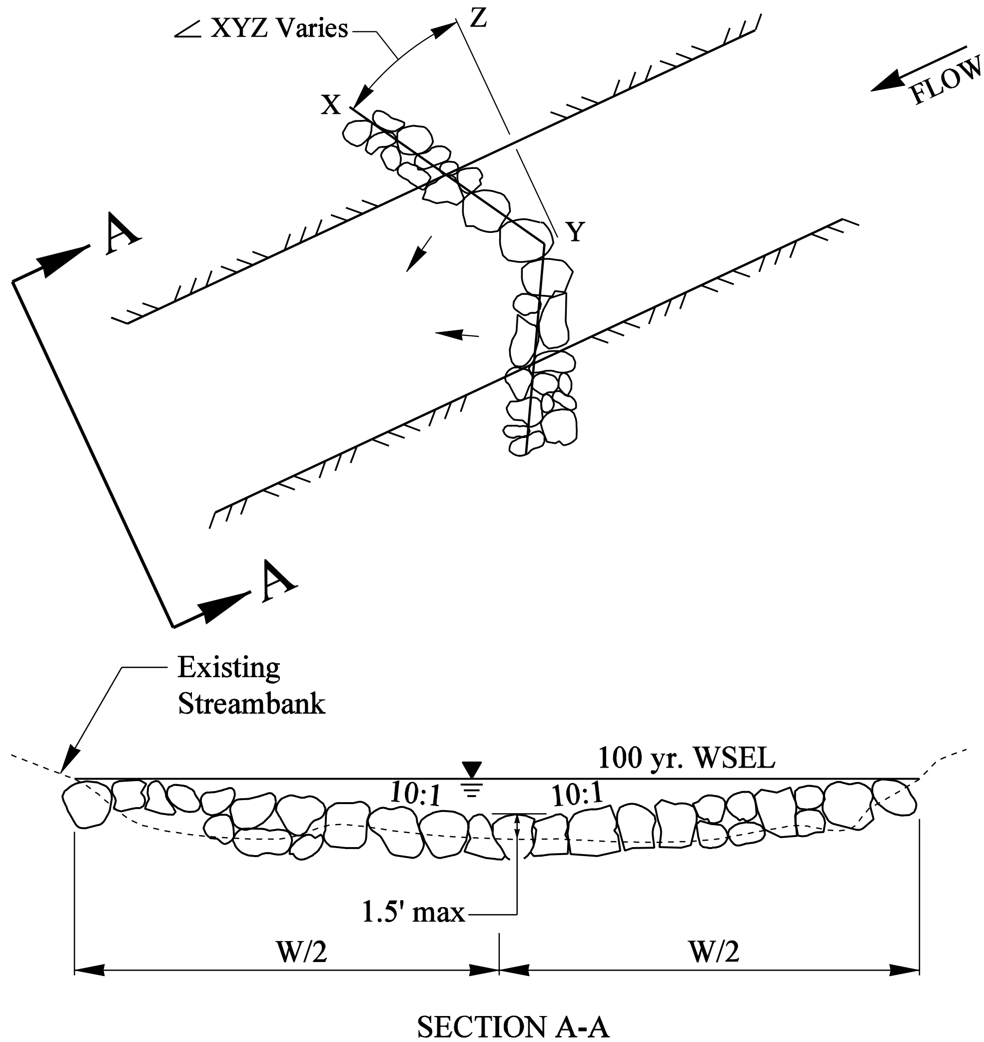
4-6.2 Drop Structures

Rock drop structures are very similar to bank barbs in their ability to redirect the flow of the river and decrease its energy. This rock structure redirects the flow towards the center of the channel and is in a V-shape with the V pointing upstream see Figure 4-6.5.2. As the river flows over the drop structure, the flow is directed perpendicular to the downstream face of the drop structure. However, because of the V-shape of the drop structure, the flow will leave the drop structure in two directions, both aiming towards the middle of the channel. Drop structures should be constructed with the XYZ angle between 20-30 degrees. Substantial scour could be experienced in the middle of the channel if angle XYZ is too large, for angles in excess of 30 degrees designer should consult the HQ Hydraulics Office.

Two considerations should be taken into account when designing a drop structure: the size of rock and its placement. The procedure for determining the size of rock needed for a drop structure is the same procedure used for river barbs. As a general rule, the size of rock used in the structure should be larger than the size of rocks existing in the bed of the channel. As for the placement of the rock the longest axis of the rock should be pointed upstream. Care should be taken in the height of the drop. The height of the structure should not exceed 1.5 feet (0.5 m) and may be restricted dependent on the species of fish present in the stream. See Chapter 7 or your project biologist for more details. If the drop is too high, a scour hole will form downstream of the base of the structure causing the structure to be undermined and fail.

It is also very important to bury a portion of the drop structure to provide a key into the bank and channel bottom. Similar to barbs, the existing streambank that the drop structure was tied into will erode, if the flow of water is allowed to get behind the key. Specific dimensions of the rock drop structure will be dependent on the river reach of interest. The designer should contact the Regional Hydraulic Engineer or HQ Hydraulics Office for design guidance.

Rock drop structures provide similar benefits as river barbs. In addition to decreasing the energy in the flow and redirecting flow, drop structures like barbs provide some protection for bridge abutments since it is a very effective river training technique.



Drop Structure Plan and Cross Section Views

Figure 4-6.5.2

Drop structures should be considered when there is a meander propagating toward a bridge. In this case, the river could get behind the bridge abutments and take out the approach fills to the bridge. Unfortunately, meander traits such as location and sinuosity are unpredictable, so unless the bridge spans the entire flood plain, there is no guarantee that the meandering river will not impact the bridge abutment. A drop structure is suitable for this situation because it spans the entire channel and can provide redirection of flow regardless of the direction the intercepted flow is heading.

A barb would not be as effective in this situation because if the flow was at an angle to the streambank, the barb would intercept very little of the flow and thus provide very little redirection. In most cases, the use of drop structures should be limited to smaller, narrow rivers and overflow channels for constructability and permitting reasons. Permitting agencies may not allow construction equipment within the floodway. If the river is too wide, it would be extremely difficult, if not impossible, to set the rocks in the center of the channel with equipment stationed along the bank. The use of drop structures is also discouraged in rivers with large bed load. This structure spans the entire channel and can be damaged when struck by large rocks or woody debris.

4-6.3 Riprap Bank Protection

Riprap bank protection is a layer of either spalls, light loose, or heavy loose riprap placed to stabilize the bank and limit the effects of erosion. Riprap is a flexible channel lining that can shift as the bank changes since the rocks are loose and free to move. Rigid channel linings are generally not recommended for the same reasons that flexible linings are recommended. If rigid linings are undermined, the entire rigid lining as a whole will be displaced increasing the chances of failure and leaving the bank unprotected. Riprap rock encased in grout is an example of a rigid channel lining.

There is disadvantages to using riprap bank protection. Adding riprap to the channel will create a smooth section or a path of least resistance that reduces the available volume of the channel creating higher velocities. This change will impact the channel down stream where the riprap ends causing a higher potential for erosion. Because of these downstream impacts to the channel, designer should consider if using riprap for bank protection would solve the problem or create a new problem.

Riprap bank protection is primarily used on the outside of curved channels or along straight channels when the streambank serves as the roadway embankment. Riprap on the inside of the curve is only recommended when overbank flow reentering the channel may cause scour. On a straight channel, bank protection should begin and end at a stable feature in the bank if possible. Such features might be bedrock outcroppings or erosion resistant materials, trees, vegetation, or other evidence of stability.

This section does not apply to an existing bridge or when historical evidence indicates that riprap will be needed around a new bridge. In those cases, the region should indicate this information on the Bridge Site Data Sheet (Form 235-001) and refer the riprap design to HQ Hydraulics Office. Section 4-6.3.3 provides additional guidance for scour analysis.

4-6.3.1 Riprap Sizing for Bank Protection

A design procedure for rock riprap channel linings was developed by the University of Minnesota as a part of a National Cooperative Highway Research Program (NCHRP) study under the sponsorship of the American Association of State Highway and Transportation Officials (AASHTO). The design procedure presented in this section is based on this study and has been modified to incorporate riprap as defined in the WSDOT *Standard Specifications*: Spalls, Light Loose Riprap, and Heavy Loose Riprap.

Once the designer has completed the analysis in this section, the designer should consider the certainty of the velocity value used to size riprap along with the importance of the facility. For additional guidance, designers can consult *NCHRP Report 568 Riprap Design Criteria* and *Hydraulic Engineering Circular 11 Design of Riprap Revetment.*

Manning's Formula or computer programs as previously discussed, compute the hydraulic capacity of a riprap-lined channel. The appropriate n-values are shown in Figure 4-6.3.1.

Type of Rock Lining[2]		**n (Small Channels**[1]**)**	**n (Large Channels)**
Spalls	D_{50}=0.5 ft (0.15m)	0.035	0.030
Light Loose Riprap	D_{50}=1.1 ft (0.32m)	0.040	0.035
Heavy Loose Riprap	D_{50}=2.2 ft (0.67m)	0.045	0.040

1. Small channels can be loosely defined as less than 1,500 cfs (45 m3/s).
2. See the WSDOT *Standard Specifications for Road and Bridge Construction* Sections 8-15 and 9-13.

Manning's Roughness Coefficients for Riprap (n)

Figure 4-6.3.1

Using Manning's Equation, the designer can determine the slope, the depth of flow, and the side slopes of the channel required to carry the design flow. The designer, using this information, can then determine the required minimum D_{50} stone size with equation (4-9).

$$D_{50}=C_R\, d\, S_o \qquad (4\text{-}9)$$

Where:
- D_{50} = Particle size of gradation, ft (m), of which 50 percent by weight of the mixture is finer
- C_R = Riprap coefficient. See Figure 4-6.3.2
- d = Depth of flow in channel, ft (m)
- S_o = Longitudinal slope of channel, ft/ft (m/m)
- B = Bottom width of trapezoidal channel, ft (m). See Figure 4-6.3.2

Channel	**Angular Rock 42° of Repose (0.25' ≤ D_{50} ≤ 3') (0.08m ≤ D_{50} ≤ 0.91m)**			**Rounded Rock 38° of Repose (0.25' ≤ D_{50} ≤ 0.75') (0.08m ≤ D_{50} ≤ 0.23m)**		
Side Slopes	**B/d=1**	**B/d=2**	**B/d=4**	**B/d=1**	**B/d=2**	**B/d=4**
1.5:1	21	19	18	28	26	24
1.75:1	17	16	15	20	18	17
2:1	16	14	13	17	15	14
2.5:1	15	13	12	15	14	13
3:1	15	13	12	15	13	12
4:1	15	13	12.5	15	13	12.5
Flat Bottom	12.5	12.5	12.5	12.5	12.5	12.5

Note: Angular rock should be used for new bank protection as it is better at interlocking and providing a stable slope. Rounded rock is unstable and is not recommended for new bank protection, the coefficients have only been provided to verify if native material is of sufficient size to resist erosion. Rounded rock use in new design should be limited to the channel bed region and to provide stream bed characteristics in a bottomless arch culvert.

Riprap Coefficients

Figure 4-6.3.2

4-6.3.1.1 Example 1 Riprap Sizing for Bank Protection

A channel has a trapezoidal shape with side slopes of 2:1 and a bottom width of 10ft. It must carry a Q_{25} = 1200 cfs and has a longitudinal slope of 0.004 ft/ft. Determine the normal depth and the type of riprap, if any, that is needed.

Using the process described in example 2 of section 4-2.2.1.1 and guessing a roughness coefficient for riprap from Figure 4-6.3.1 (for this example an n=0.035 was chosen for spalls), the normal depth was found to be d = 7.14ft with a velocity of V = 6.92ft/s. .

Next use Figure 4-6.3.2 to determine what type, if any, riprap is needed.

$$B/d = \frac{10\text{ft}}{7.14\text{ft}} = 1.4$$

Given a side slope of 2:1, and a calculated value of B/d = 1.4, C_R is noted to be between 16 and 14 in Figure 4-6.3.2for angular rock. It is allowable to interpolate between B/d columns.

$$D_{50} = C_R(d)S_o \qquad (4\text{-}9)$$

$$D_{50} = 15(7.14\text{ft})(0.004) = 0.43\text{ft}$$

From Figure 4-6.3.1, "Spalls" would provide adequate protection for a D_{50} of 0.5 ft or less in this channel. If the present stream bed has rock which exceeds the calculated D_{50}, then manmade protection is needed.

4-6.3.1.2 Example 2 Riprap Sizing for Bank Protection

Repeat the process using a 1 percent slope, and the designer finds:

$$d = 5.75\text{ft}$$

$$V = 9.72\text{ft/s}$$

$$B/d = 10/5.75 = 1.74\text{ft}$$

$$C_R = 14.5$$

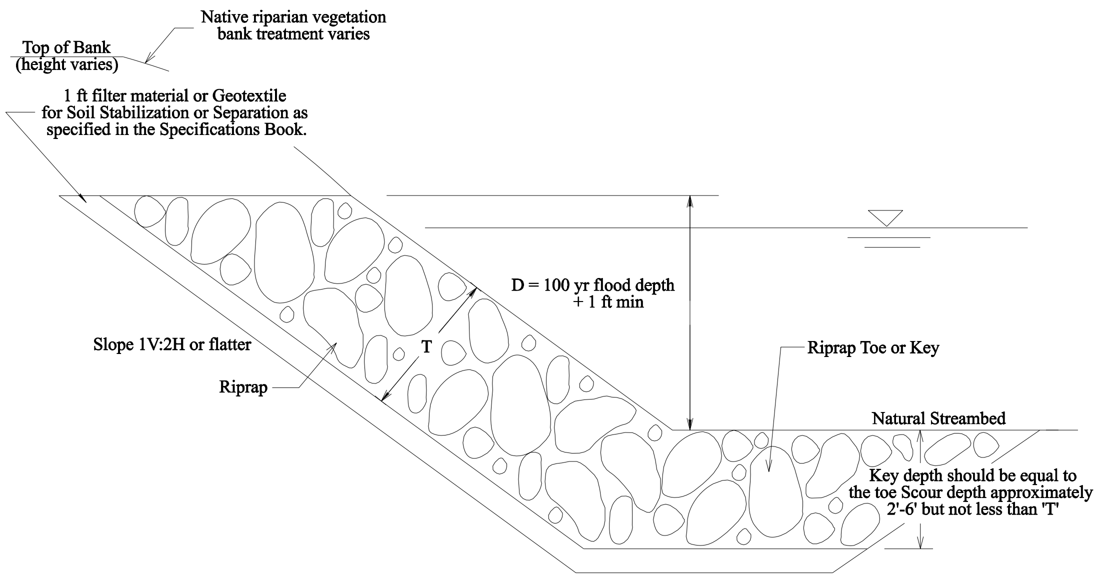
$$D_{50} = 14.5(5.75\text{ft})(0.01) = 0.83\text{ft}$$

In this case, from Figure 4-6.3.1, light loose riprap would be appropriate. Since the roughness coefficient noted in Figure 4-6.3.1 for light loose riprap is n = 0.040, the designer may recalculate the depth and velocity to get a more exact answer but this would only change the normal depth slightly and would not affect the choice of bank protection. In some cases, on very high velocity rivers or rivers that can transport large rocks downstream, even heavy loose riprap may not be adequate to control erosion and specially sized riprap may need to be specified in the contract. HQ Hydraulics Office and the Materials Lab are available for assistance in writing a complete specification for special riprap.

Once the size of riprap is determined, there are several methods in which riprap bank protection can be constructed. Two types of riprap placement including dumped rock riprap and hand-placed riprap are discussed in the following sections.

4-6.3.2 Placement of Riprap Bank Protection

Once the type of riprap has been selected from Figure 4-6.3.1, the next step is to determine the appropriate installation. Several factors affect the placement of riprap including: the type of filter material best suited for the project site, the thickness at which to place riprap, and the depth to key riprap to prevent undermining. Figure 4-6.3.3 illustrates a typical cross section of a riprap bank protection installation.



Typical Cross Section of Riprap Bank Protection Installation

Figure 4-6.3.3

The filter material acts as a transition between the native soil and the riprap, preventing the piping of fines through the voids of the riprap structure and at the same time allowing relief of the hydrostatic pressure in the soil. There are two types of filters that are used; gravel (filter blanket) or fabric (geotextile). A filter blanket may consist of a 1-foot (0.3 m) thick layer of material graded from sand to 6-inch (150-mm) gravel, (placed in layers from fine to coarse out to the riprap). Filter materials are further described in the *Standard Specifications* and *Design Manual.* In the *Standard Specifications see* Section 8-15.2 for filter blankets or Section 9-33 for geotextiles, in the *Design Manual* see Section 530 for Geotextiles. If the existing banks are similar to the filter material of sands and gravel, no filter layer maybe needed. The proper selection of a filter material is critical to the stability of the original bank material in that it aids in preventing scour or sloughing. Prior to selecting a filter material, the designer should first consult with the Project Engineer and the Region Hydraulic Engineer to determine if there is a preference. In areas of highly erodible soil (fine clay-like soils), HQ Hydraulics Office should be consulted and an additional layer of sand may be required. For additional guidance selecting the appropriate filter material see, *Hydraulic Engineering Circular No. 11*.

The thickness that riprap should be placed (shown as T in Figure 4-6.3.3 above) depends on which type of riprap was selected; quarry spalls, light loose riprap, or heavy loose riprap. Riprap thickness is 2 foot (0.6 m) for light loose riprap, 3 feet (0.9 m) for heavy loose riprap, and 1 foot (0.3 m) for quarry spalls. Care should be taken during construction to ensure that the range of riprap sizes, within each group, is evenly distributed to keep the riprap stable. Riprap is usually extended to 1 foot (0.30 m) above the 100-year flood depth of the water as shown in Figure 4-6.3.3, however if severe wave action is anticipated it should extend further up the bank.

The designer and construction inspectors must recognize the importance of a proper toe or key at the bottom of any riprap bank protection. The toe of the riprap is placed below the channel bed to a depth equaling the toe scour depth. If the estimated scour is minimal, the toe is placed at a depth equivalent to the thickness of the riprap and helps to prevent undermining. Without this key, the riprap has no foundation and the installation is certain to fail. Where a toe trench cannot be dug, the riprap should terminate in a stone toe at the level of the streambed. A stone toe (a ridge of stone) placed along steep, eroding channel banks is one of the most reliable, cost effective bank stabilization structures available. The toe provides material, which will fall into a scour hole and prevent the riprap from being undermined. Added care should be taken on the outside of curves or sharp bends where scour is particularly severe. The toe of the bank protection may need to be placed deeper than in straight reaches.

4-6.3.3 Scour Analysis for Bridges and Three Sided Culverts

Bridge scour is erosion around a bridge pier or abutment caused by the river or stream. If this type of damage is not prevented or repaired, it could cause catastrophic failure to the bridge. The typical repair for this type of damage is to place large rocks around the pier. Projects such as these can be difficult to permit because they involve placing equipment and materials in environmentally sensitive areas. Per section 1-2 of Chapter 1 of this manual, it is the responsibility of the HQ Hydraulics Office to perform all bridge scour analysis, including three sided culverts. The purpose of this section is to define scour as well as explain when an analysis maybe required and by what standards FHWA requires for a scour analysis. Also listed below is what information HQ Hydraulics requires from PEO's in order to perform a scour analysis.

Since any bridge placement within a waterway is considered a potential scour hazard, a scour analysis is required for all new bridges as well as culverts and other structures under the roadway where the amount of fill is less than half the structure opening. As conditions change at an existing site or are noted scour critical by the HQ Bridge or Hydraulics office, scour conditions may need to be re-evaluated. Once it is determined that a scour analysis is required, the region must provide the following information to the HQ Hydraulics Office in order to complete the river backwater analysis.

1. Contour information as described in item 1 in section 4-5.
2. Any proposed channel alterations including the placement of LWD components.
3. Bridge or culvert information including: pictures, dimensions, elevations, OHWM, direction of flow, and any fish passage issues.
4. Soil bearing information from the Geotech/Materials Lab.
5. Soil type and gradation of the stream (D50 and D90 values).
6. The amount of unstable material that will need to be removed and replaced.
7. Debris history from the region maintenance office to determine the vertical clearance.

The minimum requirements for a scour analysis are set by the FHWA, which requires that all bridges be designed to resist scour from a 100-year event and be checked against a 500-year event. A complete scour evaluation includes all piers and abutments in the channel migration zone. If a consultant completes the analysis; then a report of the analysis must be sent to both HQ Hydraulics and Bridge Preservation Office's for review and approval. The consultant should contact the HQ Hydraulics Office for scour report guidelines. The 100 and 500-year flows and water surface elevations must be included on the bridge plan sheets. See the Hydraulic Report Outline in Chapter 1 for further guidance on what should be on the plan sheets.

4-6.4 Engineered Log Jams and Large Woody Debris

Streambank erosion can be controlled by slowing down the water velocity and reducing the hydraulic shear. This can be achieved by adding roughness to the channel which in turn increases the friction in the channel. Such roughness can be introduced by installing Large Woody Debris (LWD) in the channel and along the banks. Also used are, Engineered Log Jams (ELJ), a collection of LWD that redirect flow and provide stability to a streambank.

Large Woody Debris (LWD) may be a single log or a small group of logs with the root wads still attached. As previously mentioned, LWD is typically used as a roughness feature however, when positioned properly, LWD can trap sediment which enables vegetation to establish itself ultimately stabilizing actively eroding banks. LWD can also be used to enhance wild life by; dissipating flow energy resulting in improved fish migration, as well as providing over head cover for fish and basking/perching sites for reptiles and birds. LWD can adversely affect the channel's hydraulic characteristics if placed properly; contact the HQ Hydraulics Office for further design guidance.

Engineered Log Jams (ELJ) are in-stream structures composed mainly Large Woody Debris (LWD) that direct flow and may provide stability to a streambank to protect it from erosional forces. ELJ has become increasingly popular as bank protection because they integrate fish-habitat restoration with bank protection. ELJ can either be unanchored or anchored in-place using man-made materials. Prior to designing and constructing an ELJ as a bank protection technique, it is important to understand the existing physical characteristics and geomorphic processes present at a potential site. ELJ are considered experimental and as such HQ Hydraulics is responsible for ELJ design, see section 1-2 of this manual.

4-7 Downstream Analysis

A downstream analysis identifies and evaluates the impacts, if any, a project will have on the hydraulic conveyance system downstream of the project site. All projects that propose to discharge stormwater offsite and meet the requirements below are required to submit a downstream analysis report as part of the Hydraulics Report, see the Hydraulic Report Outline in Chapter 1.

- Projects that add 5,000 square feet or more of impervious surface area.
- Project sites where known problems indicate there may be impacts on the downstream system.
- Projects that add less than 5,000 square feet of new impervious surface if the stormwater discharges into, or is within 300 feet of, a class 1 or 2 stream.

- Projects that add less than 5,000 square feet of new impervious surface, if the stormwater discharges into or is within 300 feet of a class 3 or 4 stream or an ephemeral stream.

Additionally, any outfall (either man-made or natural) where stormwater from WSDOT highways is conveyed off the ROW must be entered into the WSDOT Outfall Database. See Appendix 1-3 section 2.5 of this manual for further guidance.

4-7.1 Downstream Analysis Reports

At a minimum, the analysis must include the area of the project site to a point one-quarter mile downstream of the site, and upstream to a point where any backwater conditions cease. The results of the analysis must be documented in the project Hydraulic Report. Potential impacts to be assessed in the report also include, but are not limited to: changes in peak flow, changes in flood duration, bank erosion, channel erosion, and nutrient loading changes from the project site. The analysis is divided into three parts that follow sequentially:

1. Review of Resources.
2. Inspection of drainage conveyance systems in the site area.
3. Analysis of offsite effects.

4-7.2 Review of Resources

The designer reviews available resources to assess the existing conditions of the drainage systems in the project vicinity. Resource data commonly includes aerial photographs, area maps, floodplain maps, wetland inventories, stream surveys, habitat surveys, engineering reports concerning the entire drainage basin, and any previously completed downstream analyses. All of this information should encompass an area one-quarter of a mile downstream of the project site discharge point. The background information is used to review and establish the existing conditions of the system. This base-line information is used to determine whether the project will improve upon existing conditions, have no impact, or degrade existing conditions if no mitigating measures are implemented. WSDOT Region hydraulic and environmental staff will be able to provide most of this information. Other sources of resource information include the Washington Department of Ecology, the Washington Department of Fish and Wildlife, and local agencies.

4-7.3 Inspection of Drainage Conveyance System

The designer must inspect the downstream conveyance system and identify any existing problems that might relate to stormwater runoff. The designer will physically inspect the drainage system at the project site and downstream for a distance of at least one-quarter mile. The inspection should include any problems or areas of concern that were noted during the resource review process or in conversations with local residents and the WSDOT Maintenance Office. The designer should also identify any existing or potential conveyance capacity problems in the drainage system, any existing or potential areas where flooding may occur, any existing or potential areas of extensive channel destruction erosion, and existing or potential areas of significant destruction of aquatic habitat (runoff treatment or flow control) that can be related to stormwater runoff. If areas of potential and existing impacts related to project site runoff are established, actions must be taken to minimize impacts to downstream resources.

4-7.4 Analysis of Off Site Affects

This final step analyzes information gathered in the first two steps of the downstream analysis. It is necessary to determine if construction of the project will create any problems downstream or make any existing problems worse. The designer must analyze off-site effects to determine if corrective or preventive actions that may be necessary. Designers should consult the HRM for further guidance on the design flow. In some cases, analysis of off site effects may indicate that no corrective or preventive actions are necessary. If corrective or preventive actions are necessary, the following options must be considered:

Design the onsite treatment and/or flow control facilities to provide a greater level of runoff control than stipulated in the minimum requirements in Chapter 2 of the *HRM*.

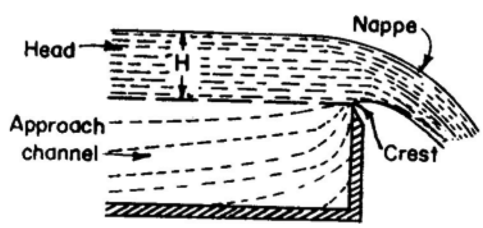
Take a protective action separate from meeting Minimum Requirements 5 and 6 for runoff treatment and flow control. In some situations, a project will have negative impacts even when the minimum requirements are met; for example, a site where the project discharges runoff into a small closed basin wetland even though a detention pond was installed to comply with Minimum Requirement 6. The total volume of runoff draining into the wetland will change, possibly affecting habitat and plant species in the area. If a situation is encountered where there will be downstream impacts resulting from the project, the corrective action must be applied to the project based on a practicability analysis.

Apply the no action at 0 percent improvement option for runoff treatment or flow control. The no action option treats less than 100 percent of the new impervious surface area for runoff treatment and/or flow control. This option would be applied only if the downstream system has been listed as an exempt system based on Minimum Requirement 6, or an Explanation of Non-practicability has been addressed. Under these circumstances, the designer should contact Region Hydraulics or Environmental Staff to determine the best corrective action

4-8 Weirs

The weirs described in this section are primarily used for measuring flow rate in irrigation channels. Designers should consult the *Highway Runoff Manual*, Chapter 5 for further guidance on weirs for other uses. Measurement of flow in channels can be difficult because of the non-uniform channel dimensions and variations in velocities across the channel. Weirs allow water to be routed through the structure of known dimension, permitting flow rates to be measured as a function of depth of flow through the structure.

The opening of a weir is called a notch; the bottom edge is the crest; and the depth of flow over the crest is called the head. The overflowing sheet of water is known as the nappe.



Sharp Crested Weir

Figure 4-8.1

Sharp crested weirs cause the water to spring clear of the crest providing an accurate measurement for irrigation channels, see Figure 4-8.1. There are other types of weirs, however sharp crested weirs are the focus of this section.

The common types of sharp crested weirs are rectangular, V-notch and compound. These three weirs are the focus of this section. All three weirs require a stilling pool or approach channel on the upstream side to smooth out any turbulence and ensure that the water approaches the notch slowly and smoothly. For accurate measurements the specification is that the width of the approach channel should be 8 times the width of the notch and it must extend upstream for 15 times the depth of flow over the notch.

4-8.1 Rectangular Weirs

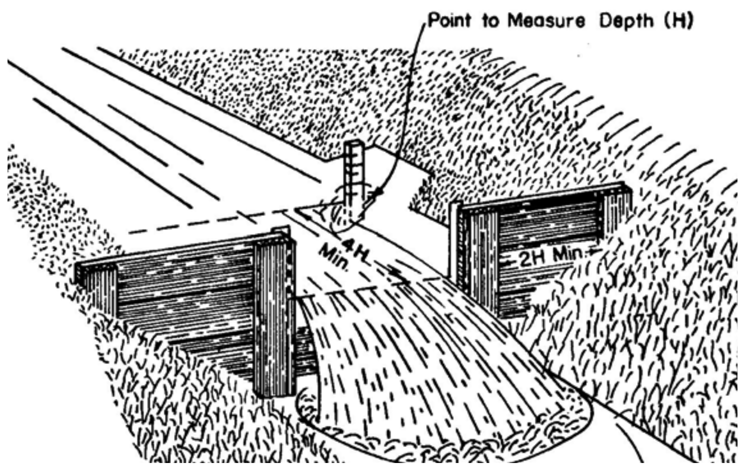
Rectangular weirs are the oldest type of weirs in use. It is recommend for higher discharge rates, above 10cfs and not recommended for low discharge rates (less than 10 cfs) or when there is a wide range of flow. The flow rate measurement in a rectangular weir is based on the Bernoulli Equation principles and is expressed as:

$$Q = 3.33H^{3/2}(L - 0.2H) \qquad (4\text{-}10)$$

Where: Q = Discharge in cfs second neglecting velocity of approach

L = the length of weir, in feet

H = Head on the weir in feet measured at a point no less than 4 H upstream from the weir.



Rectangular Weir

Figure 4-8.1.1

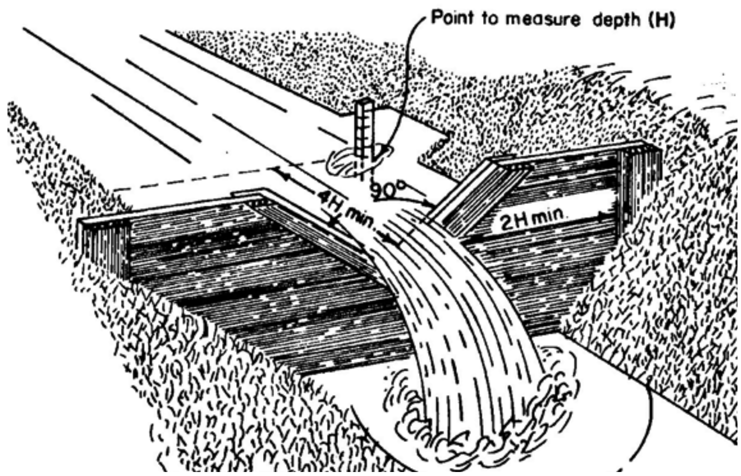
4-8.2 V-Notch Weirs

V-notch weirs measure low discharges, less than 10 cfs, more accurately than rectangular weirs. The V-notch is most commonly 90° opening with the sides of the notch inclined 45° with the vertical. Since the V-notch has no crest length, much smaller flows are represented by a given head than for a rectangular weir.

The discharge equation used for V-notch weirs is:

$$Q = 2.52H^{2.47} \qquad (4\text{-}10)$$

Where: H = Vertical distance in feet between the elevation of the vortex or lowest part of the notch and the elevation of the weir pond.



V-notch Weir

Figure 4-8.1.2

4-8.3 Trapezoidal or Cipoletti Weirs

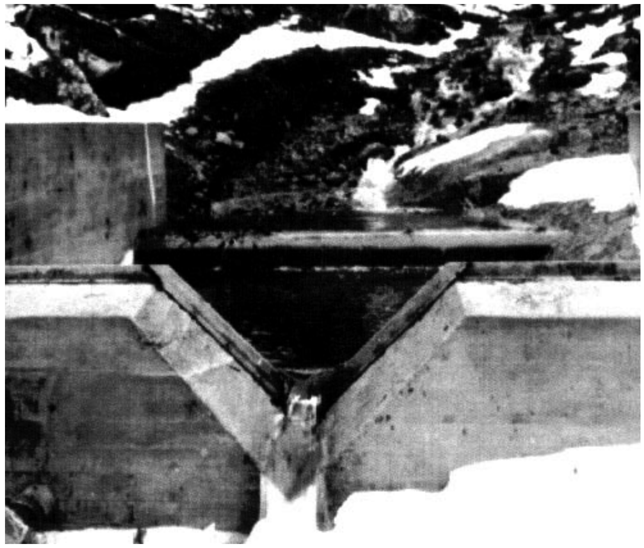
A trapezoidal weir is a combination rectangular weir with the sides sloped to compensate for end contractions. This shape permits good measurements in streams with a wide range of flows as the sloped section is sized for low flow conditions while larger flows are measured with the rectangular weir. The discharge over a trapezoidal weir is calculated by simply applying the standard discharge equation for each segment of the weir to the head on that segment of the weir. The total discharge is then the sum of the discharges of each of the two segments of the weir as shown below: Cipolletti weirs are trapezoidal with 1:4 slopes to compensate for end contraction losses

$$Q = 3.367 L H^{1.5} \tag{4-11}$$

Where: Q = Discharge in cfs

L =width of the bottom section of the weir in feet

H =head above the horizontal crest in feet



Trapezoidal Weir

Figure 4-8.1.3

Appendix 4-1 Manning's Roughness Coefficients (n)

I. Closed Conduits

A. Concrete pipe 0.010-0.011

B. Corrugated steel or Aluminum circular pipe or pipe-arch:

1. 2 2/3 × 1/2 in. Annular Corrugations, treated or untreated 0.022-0.027

2. 2 2/3 × 1/2 in. Helical Corrugations

a. Plain or Protective Treatments 1

(1) 18 inch diameter and below 0.013

(2) 24 inch diameter 0.015

(3) 36 inch diameter 0.018

(4) 48 inch diameter 0.021

(5) 60 inch diameter 0.022

(6) 72 inch diameter and above 0.024

b. Protective Treatments 2 or 4[1]

(1) 18 inch diameter and below 0.012

(2) 24 inch diameter 0.014

(3) 36 inch diameter 0.017

(4) 48 inch diameter 0.020

(5) 60 inch diameter 0.021

(6) 72 inch diameter and above 0.023

c. Protective Treatments 5 or 6[1]

(1) All diameters 0.012

3. 3 × 1 in. Annular Corrugations, treated or untreated 0.027-0.028

1. Treatments 3, 4 and 6 are no longer available and appear only for reference.

4. 3 × 1 in. Helical Corrugations
   a. Plain or Protective Treatments 1 or 3[1]
      (1) 54 inch diameters and below 0.023
      (2) 60 inch diameter 0.024
      (3) 72 inch diameter 0.026
      (4) 78 inch diameter and above 0.027
   b. Protective Treatments 2 or 4[1]
      (1) 54 inch diameters and below 0.020
      (2) 60 inch diameter 0.021
      (3) 72 inch diameter 0.023
      (4) 78 inch diameter and above 0.024
   c. Protective Treatments 5 or 6[1]
      (1) All diameters 0.012
5. 5 × 1 in. Annular Corrugations, treated or untreated 0.025-0.026
6. 5 × 1 in. Helical Corrugations
   a. Plain or Protective Treatments 1 or 3[1]
      (1) 54 inch diameters and below 0.022
      (2) 60 inch diameter 0.023
      (3) 66 inch diameter 0.024
      (4) 72 inch diameter and above 0.025
   b. Protective Treatments 2 or 4[1]
      (1) 54 inch diameters and below 0.019
      (2) 60 inch diameter 0.020
      (3) 66 inch diameter 0.021
      (4) 72 inch diameter and above 0.022
   c. Protective Treatments 5 or 6[1]
      (1) All diameters 0.012

1. Treatments 3, 4 and 6 are no longer available and appear only for reference.

C. Steel or Aluminum Spiral Rib Pipe 0.012-0.013

D. Structural Plate Pipe and Plate Pipe Arches 0.033-0.037

E. Thermoplastic Pipe 0.012

1. Corrugated Polyethylene, HDPE 0.018-0.025
2. Profile wall polyvinyl chloride, PVC 0.009-0.011
3. Solid wall polyvinyl chloride, PVC 0.009-0.015

F. Cast-iron pipe, uncoated 0.013

G. Steel pipe 0.009-0.011

H. Vitrified clay pipe 0.012-0.014

I. Brick 0.014-0.017

J. Monolithic concrete:

1. Wood forms, rough 0.015-0.017
2. Wood forms, smooth 0.012-0.014
3. Steel forms 0.012-0.013

K. Cemented rubble masonry walls:

1. Concrete floor and top 0.017-0.022
2. Natural floor 0.019-0.025

L. Laminated treated wood 0.015-0.017

M. Vitrified clay liner plates 0.015

II. Open Channels, Lined (Straight Alignment)

A. Concrete, with surfaces as indicated:

1. Formed, no finish 0.013-0.017
2. Trowel finish 0.012-0.014
3. Float finish 0.013-0.015
4. Float finish, some gravel on bottom 0.015-0.017
5. Gunite, good section 0.016-0.019
6. Gunite, wavy section 0.018-0.022

B. Concrete, bottom float finished, sides as indicated:

1. Dressed stone in mortar 0.015-0.017
2. Random stone in mortar 0.017-0.020
3. Cement rubble masonry 0.020-0.025
4. Cement rubble masonry, plastered 0.016-0.020
5. Dry rubble (riprap) 0.020-0.030

C. Gravel bottom, sides as indicated:

1. Formed concrete 0.017-0.020
2. Random stone in mortar 0.020-0.023
3. Dry rubble (riprap) 0.023-0.033

D. Brick 0.014-0.017

E. Asphalt:

1. Smooth 0.013
2. Rough 0.016

F. Wood, planed, clean 0.011-0.013

G. Concrete-lined excavated rock:

1. Good section 0.017-0.020
2. Irregular section 0.022-0.027

III. Open Channels, Excavated (Straight Alignment, Natural Lining)

A. Earth, uniform section:

1. Clean, recently completed 0.016-0.018
2. Clean, after weathering 0.018-0.020
3. With short grass, few weeds 0.022-0.027
4. In gravelly soil, uniform section, clean 0.022-0.025

B. Earth, fairly uniform section:

1. No vegetation 0.022-0.025
2. Grass, some weeds 0.025-0.030
3. Dense weeds or aquatic plants in deep channels 0.030-0.035

4. Sides clean, gravel bottom 0.025-0.030
5. Sides clean, cobble bottom 0.030-0.040

C. Dragline excavated or dredged:

1. No vegetation 0.028-0.033
2. Light brush on banks 0.035-0.050

D. Rock:

1. Based on design section (riprap) (see section 4-6) 0.035
2. Based on actual mean section:
   a. Smooth and uniform 0.035-0.040
   b. Jagged and irregular 0.040-0.045

E. Channels not maintained, weeds and brush uncut:

1. Dense weeds, high as flow depth 0.08-0.12
2. Clean bottom, brush on sides 0.05-0.08
3. Clean bottom, brush on sides, highest stage of flow 0.07-0.11
4. Dense brush, high stage 0.10-0.14

IV. Highway Channels and Swales With Maintained Vegetation (values shown are for velocities of 2 and 6 fps)

A. Depth of flow up to 0.7 foot:

1. Bermudagrass, Kentucky bluegrass, buffalograss:
   a. Mowed to 2 inches 0.07-0.045
   b. Length 4 to 6 inches 0.09-0.05
2. Good stand, any grass:
   a. Length about 12 inches 0.18-0.09
   b. Length about 24 inches 0.30-0.15
3. Fair stand, any grass:
   a. Length about 12 inches 0.14-0.08
   b. Length about 24 inches 0.25-0.13

B. Depth of flow 0.7-1.5 feet:

1. Bermudagrass, Kentucky bluegrass, buffalograss:

a. Mowed to 2 inches 0.05-0.035

b. Length 4 to 6 inches 0.06-0.04

2. Good stand, any grass:

a. Length about 12 inches 0.12-0.07

b. Length about 24 inches 0.20-0.10

3. Fair stand, any grass:

a. Length about 12 inches 0.10-0.06

b. Length about 24 inches 0.17-0.09

V. Street and Expressway Gutters

A. Concrete gutter, troweled finish 0.012

B. Asphalt pavement:

1. Smooth texture 0.013

2. Rough texture 0.016

C. Concrete gutter with asphalt pavement:

1. Smooth 0.013

2. Rough 0.015

D. Concrete pavement:

1. Float finish 0.014

2. Broom finish 0.016

3. Street gutters 0.015

E. For gutters with small slope, where sediment may accumulate, increase above values of n by 0.002

VI. Natural Stream Channels

A. Minor streams (surface width at flood stage less than 100 ft):

1. Fairly regular section:

a. Some grass and weeds, little or no brush 0.030-0.035

b. Dense growth of weeds, depth of flow materially greater than weed height 0.035-0.05

c. Some weeds, light brush on banks 0.035-0.05

d. Some weeds, heavy brush on banks 0.05-0.07

e. Some weeds, dense willows on banks 0.06-0.08

f. For trees within channel, with branches submerged at high stage, increase all above values by 0.01-0.02

2. Irregular sections, with pools, slight channel meander; increase values given in 1a-e above 0.01-0.02

3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:

   a. Bottom of gravel, cobbles, and few boulders 0.04-0.05

   b. Bottom of cobbles, with large boulders 0.05-0.07

B. Flood plains (adjacent to natural streams):

1. Pasture, no brush:

   a. Short grass 0.030-0.035

   b. High grass 0.035-0.05

2. Cultivated areas:

   a. No crop 0.03-0.04

   b. Mature row crops 0.035-0.045

   c. Mature field crops 0.04-0.05

3. Heavy weeds, scattered brush 0.05-0.07

4. Light brush and trees:

   a. Winter 0.05-0.06

   b. Summer 0.06-0.08

5. Medium to dense brush:

   a. Winter 0.07-0.11

   b. Summer 0.10-0.16

6. Dense willows, summer, not bent over by current 0.15-0.20

7. Cleared land with tree stumps, 100 to 150 per acre:
    a. No sprouts 0.04-0.05
    b. With heavy growth of sprouts 0.06-0.08
8. Heavy stand of timber, a few down trees, little under-growth:
    a. Flood depth below branches 0.10-0.12
    b. Flood depth reaches branches 0.12-0.16

C. Major streams (surface width at flood stage more than 100 ft): Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values of n may be somewhat reduced. Follow recommendation in publication cited if possible. The value of n for larger streams of most regular section, with no boulders or brush, may be in the range of 0.028-0.033

Reference: UT Chow “Open Channel Hydraulics” for complete tables and photographs